The Effects of Base Type on Modeling of LTPP CRC Sections

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ABSTRACT

Newer continuously reinforced concrete (CRC) pavements in South Dakota have exhibited undesirable levels of transverse cracking. This poor performance was not expected under the current recommended design practices. Long Term Pavement Performance (LTPP) CRC pavement data, previously analyzed by others, also could not account for the cracking. To seek an explanation the original LTPP CRC data was reanalyzed using a more thorough approach. Using multiple regression techniques on selected LTPP CRC data sets of comparable accuracy, consequential conclusions can be drawn once the base types are separated into subsets in the database. A similar model was developed for newer CRC pavements in South Dakota and showed the same response variables as the LTPP subset for granular bases. Significant correlation was found between cracks and steel depth, cracks and steel size, and cracks and pavement thickness, such that recommendations are made for South Dakota to decrease the steel depth and decrease the steel size and percentage. The nominal top size of the coarse aggregate was also found to be a significant contributor to crack width with a shift from 3/4" to 1-1/2" resulting in a reduction in crack width and a much slower development of cracking over time.

INTRODUCTION

The State of South Dakota has 545 2-lane miles of continuously reinforced concrete (CRC) pavements. The first CRC sections were constructed in 1962, and South Dakota has continued throughout the years adding to this initial length and reconstructing other roadway sections into CRC. Over the years, South Dakota has designed these CRC sections according to recommendations and guidelines that were current at the time. As a result, the designs have varied with respect to pavement depth, steel ratio, steel spacing, and placement, and for the most part, these CRC sections have performed acceptably. A typical plan view of a CRC section is shown in Figure 1.

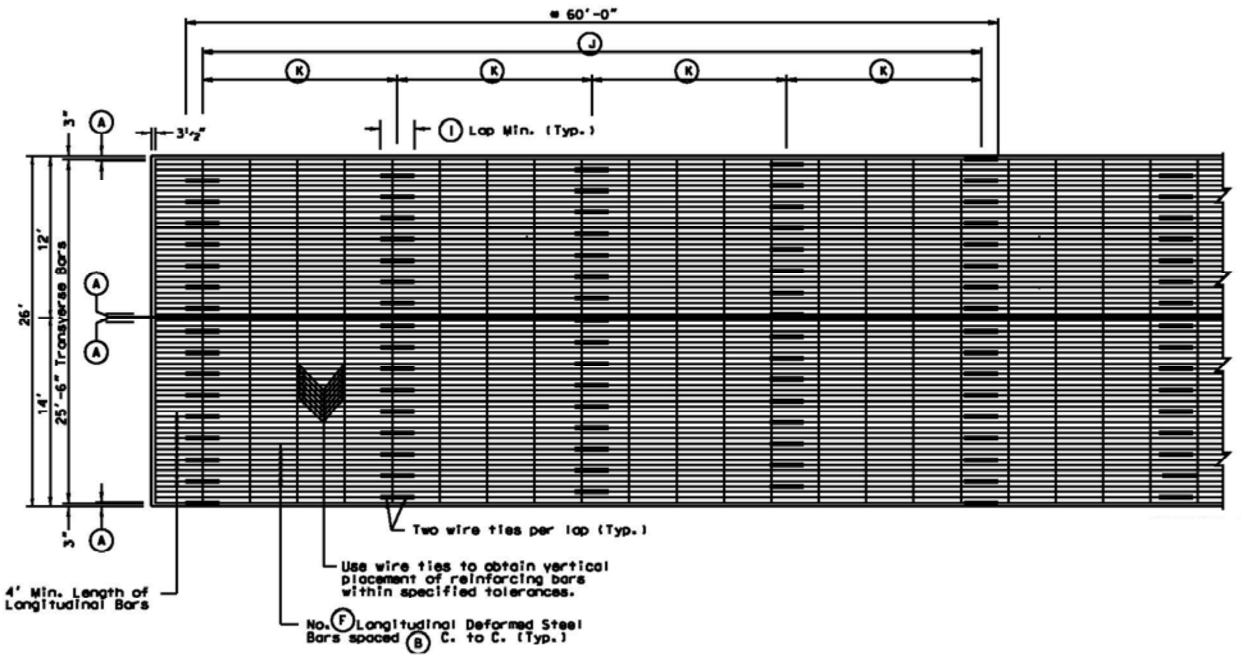

FIGURE 1 Typical CRC pavement layout.

Recently however, South Dakota's newer CRC pavements, built since 1995, have begun to exhibit undesirable performance in the form of more transverse cracks per longitudinal length with excessive y-cracking, cluster cracking and network cracking as well as wider crack widths. In an effort to gain insight into the most critical of the numerous variables that may be contributing to this undesirable performance in South Dakota's newer CRC pavements, data on 85 Long Term Pavement Performance (LTPP) test sections from 29 states were analyzed using multiple regression techniques.

EXAMINATION OF AN EARLIER ANALYSIS OF CRC SECTIONS FROM THE LTPP DATABASE

An initial reading of the report, *Preliminary Evaluation of LTPP Continuously Reinforced Concrete (CRC) Pavement Test Sections*[(1)], issued in July, 1999 proved somewhat disheartening as the authors had conducted such an analysis without obtaining any statistically significant results and presented conclusions based on a direct comparison of the ten best performing sections versus the 13 worst performers in the database. They concluded that the worst performers had the following common characteristics:

- Larger crack spacing
- Greater depth to reinforcement
- High value of mean slab thickness
- Low values of elastic moduli for slab and base layer
- Low k-value for subgrade

The well performing sections, on the other hand, had the following in common:

- Smaller crack spacing
- Lower IRI (selection criteria)
- Shallow depth to reinforcement
- Thinner and stronger slab
- Stiffer base and subgrade layers

Unfortunately, these insights are of limited value with respect to newer CRC performance in South Dakota, as relatively smaller crack spacing of 1.5-4' are typical of South Dakota's newer sections indicating the possibility that the primary source of the problem may lie outside the realm of the variables examined in the LTPP analysis except for the reinforcement depth and slab thickness. The primary performance problem with CRC pavements nationwide has been ascribed to too great a distance between transverse cracks resulting in excessive crack widths and accelerated distress. The increase in design slab thickness from 8" to 10-11" in some of the newer CRC pavements may also be a factor but thinner slabs exhibiting the same cracking patterns do not support this observation.

RE-ANALYSIS OF CRC SECTIONS FROM THE LTPP DATABASE

In an effort to glean any useable results possible from the LTPP database, a preliminary assessment of the original data analysis methods was made. The first potential shortcoming of the 1999 analysis became obvious comparing crack spacing derived from both manual and Pavement Distress Analysis System (PADIAS) surveys. The PADIAS data, based on interpretation of 35 mm photographs tends to be significantly different than the manual data and frequently has a lower value. The LTPP researchers used the highest value, either PADIAS or manual, for their analysis, which presupposes at least equivalent accuracy for the PADIAS approach. All sections, which had only PADIAS crack spacing values, were excluded from the database used for the new analysis. Next, the LTPP researchers assumed that the initial cracking response from the individual test sections could all be analyzed as a single dataset with no significant differences in cracking due to the base type for each section. Figure 2[(2)] illustrates the significantly different frictional resistance response from various base types, especially cement stabilized ones where the adhesion between the base and Portland cement concrete is so strong that large compressive forces build up, resulting in tensile stresses when the base acts to resist movement by the slab during expansion or contraction. The other base types yield at much lower levels of stress.

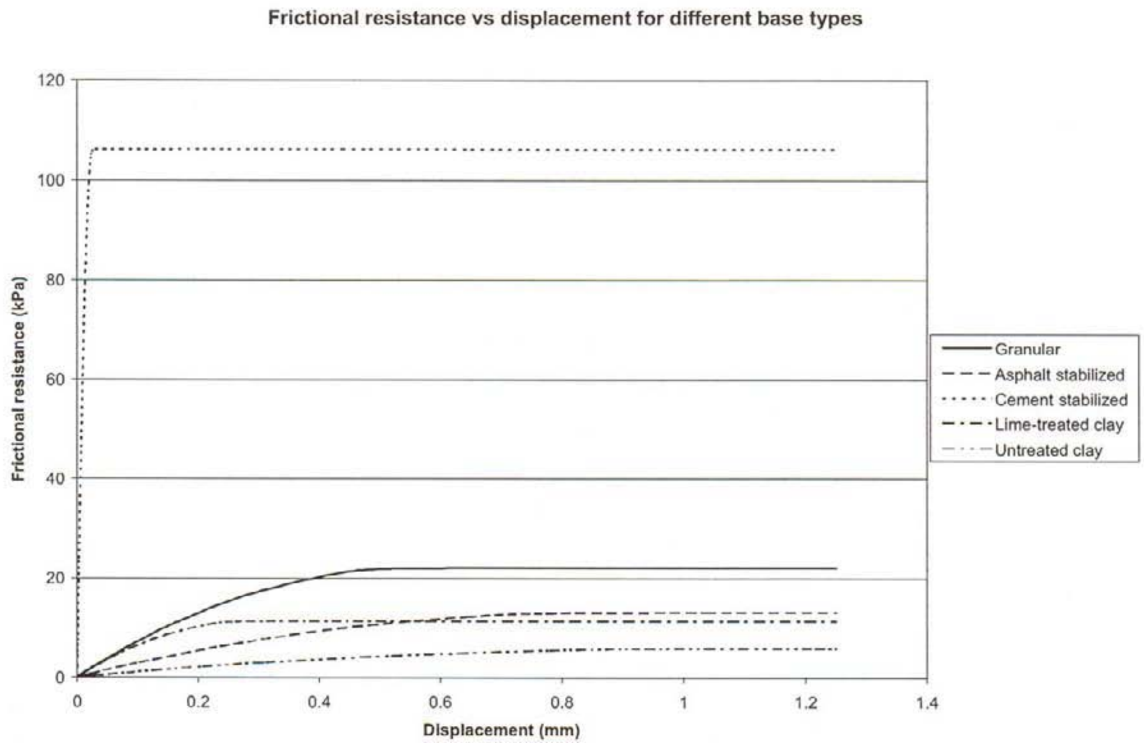

FIGURE 2 Typical friction resistance relationships.

The database was separated into granular, asphalt-treated aggregate, cement-treated aggregate and cement/lime-stabilized soil subgroups and these were analyzed for significant independent variables contributing to transverse cracking in the LTPP CRC test sections. In addition, the dependent variable used for the analysis was not the average crack spacing in feet for each 500' section but was instead, the number of cracks counted in each section. Using the crack spacing created a problem with the analysis due to the fact that, in that form, the values were inverse to several of the independent variables. The results of these analyses are shown in Tables 1 and 2 and in Figures 3 through 6 and demonstrate the fact that statistically significant relationships between cracking and several independent variables do exist ($p<.05$) but the response to each factor is critically dependent on the base type. An additional factor in the analysis of each subgroup was the number of sections with sufficient available data inclusion. One section (46-5025) in the granular subgroup was mislabeled as a granular base when it actually had a lime-treated gravel cushion. It was included with the soil cement subgroup. Two sections, one each in the granular and ACM subgroups were excluded as extreme outliers with the result being an improved significance to the models with no substantial change in factors. The comparison of average values across all base types allows clarification of why some factors may be significant for one base type but not another.

The most interesting aspect of the statistical results is the verification that the cement-aggregate subgroup demonstrated a strikingly different model than either the granular or AC section models. Both the granular and AC models showed expected relationships with slab thickness and % steel. Increasing slab thickness resulted in a decrease in cracking whereas increasing the % steel correlated with an increase in cracking. The cement-aggregate sections, on the other hand, displayed exactly the opposite situation with an increase in slab thickness

TABLE 1 LTPP CRC Test Section Variables

Factor	Base Type							
	Granular N=10		ACM n=32		SC, LT, CT n=6		CAM n=17	
	Average	S.. D.	Average	S.. D.	Average	S.. D.	Average	S.. D.
Number of Cracks	116.50	30.80	146.66	51.11	173.33	69.23	133.41	52.83
Age (years)	17.5	3.92	15.66	6.06	18.5	3.39	14.23	8.10
Slab Thickness (in.)	9.19	1.59	8.74	1.20	8.10	0.51	8.95	0.95
% Steel	0.613	0.049	0.593	0.084	0.638	0.058	0.597 n=16	0.074
Steel Depth (in.)	3.44 n=9	0.46	4.07 n=29	0.76	3.30	0.65	3.78 n=16	0.73
Steel Ratio	0.386 n=9	0.084	0.463 n=29	0.052	0.407	0.075	0.424 n=16	0.080
Lap Length (ft.)	19.0 n=8	1.77	28.27	16.92	22.00	6.93	21.00 n=16	12.03
Longitudinal Bar Diameter (in.)	0.663	0.103	0.678	0.077	0.638	0.020	0.685 n=16	0.065
Transverse Bar Diameter (in.)	0.644 n=7	0.253	0.514	0.041	0.562	0.087	0.535 n=11	0.081
Longitudinal Spacing (mm)	180.97 n=9	59.77	182.00	25.40	156.63	6.56	175.40 n=16	26.09
Transverse Spacing (mm)	999.12 n=6	307.57	892.61	188.29	944.92 n=5	198.74	1140.76 n=11	368.63
Temperature Difference (^{O} C)	11.69	1.27	13.01	1.57	12.88	1.64	12.55	1.66
Concrete Modulus (GPa)	32.31	7.79	31.88 n=12	5.84	27.05	6.63	31.44 n=16	6.37
Base Thickness (mm)	182.90	124.81	104.38	34.34	110.00	37.72	129.18	35.18
Base Modulus (GPa)	5.96	1.03	6.43	1.54	5.98	1.67	6.42 n=16	1.63
Subgrade k Value (mPa/mm)	57.70	23.07	89.84	38.48	59.33	32.73	80.25 n=16	38.09
Precipitation (mm)	367.4	308.31	184.63	245.57	232.67	274.25	215.53	250.78
Freeze Index (^{O} C days)	1015.10	155.20	892.25	350.93	1013.83	401.00	968.65	407.90
ESAL's (18 kip)	16,004	19484	7422	10951	9938	15019	7620	5896

ACM-Dense-graded AC Hot Mix; SC-Soil Cement; CT-Cement-treated Subgrade Soil; LT-Lime-treated Subgrade Soil; CAM-Cement-Aggregate Mixture

TABLE 2 Statistical Model Results for LTPP CRC Pavement Test Sections

Factor	Base Type							
	Granular n=8	P value/ F	ACM n=28	P value/ F	SC, LT, CT n=6	p value/ F	CAM n=16	P value/ F
Model R^2	0.993	0 254.3	0.817	0 12.74	0.995	0.007 136.6	0.929	0 19.48
	Coefficient	p value	Coefficient	p value	Coefficient	p value	Coefficient	p value
Constant	-365.103	0	3520.53	0	2093.52	.002	-204.461	0.045
Slab Thickness	-21.79	0.002	-444.04	0	-137.508	0.003	21.754	0.008
% Steel	861.60	0	339.37	0	-1201.68	0.003	-505.302	0
Steel Depth	35.29	0	917.22	0	(-2.297)	(0.835)	(10.55)	(0.310)
Steel Ratio	(-359.29)	(0.243)	-7898.03	0	(-18.02)	(0.837)	(97.66)	(0.345)
Lap Length	(-0.681)	(0.522)	(0.006)	(0.990)	(0.782)	(0.705)	1.556	0.004
Placement	(1.799)	(0.375)	31.17	0.005	-	-	-31.98	0.047
Temperature Difference	(-1.122)	(0.536)	11.87	0.001	(7.082)	(.086)	28.463	0
Concrete Modulus	(0.066)	(0.756)	(6.5) (n=11)	(0.038)	(-1.147)	(0.147)	2.3	0.025
Base Thickness	(-0.007)	(0.798)	0.554	0.001	(-0.082)	(0.729)	(-0.096)	(0.764)
Subgrade k Value	(-0.115)	(0.117)	(-0.042)	(0.745)	-0.662	0.024	(-0.039)	(0.813)

ACM-Dense-graded AC Hot Mix; SC-Soil Cement; CT-Cement-treated Subgrade Soil; LT-Lime-treated Subgrade Soil; CAM-Cement-Aggregate Mixture

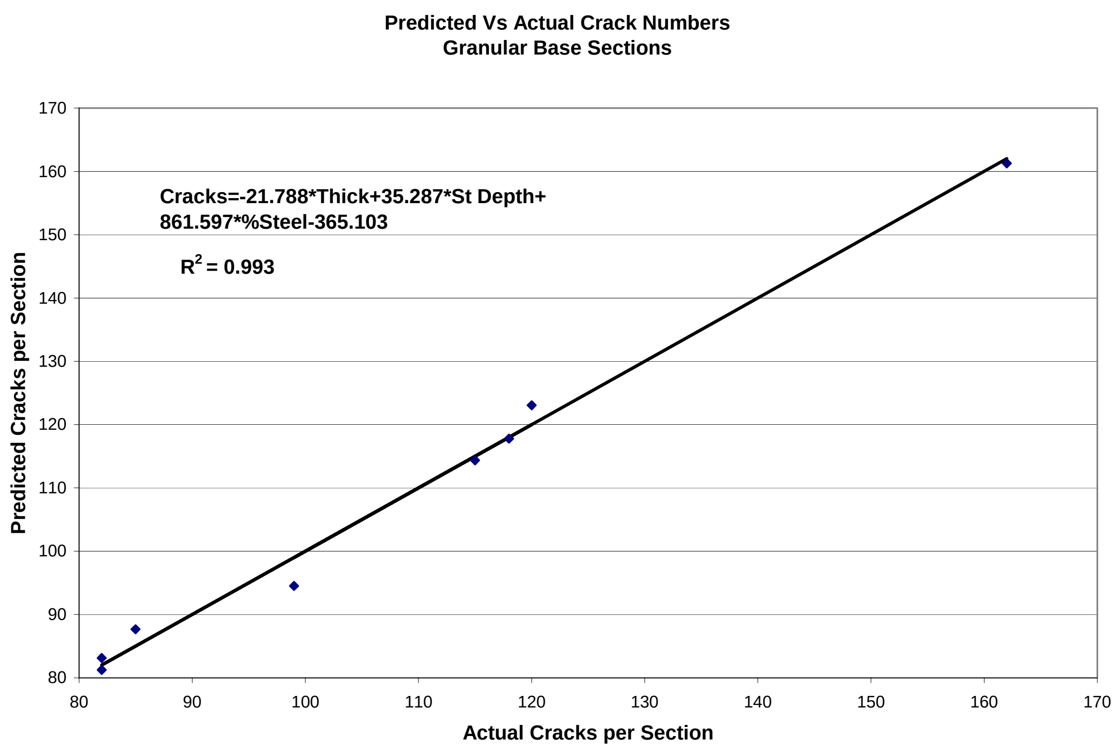

FIGURE 3 Granular sections model.

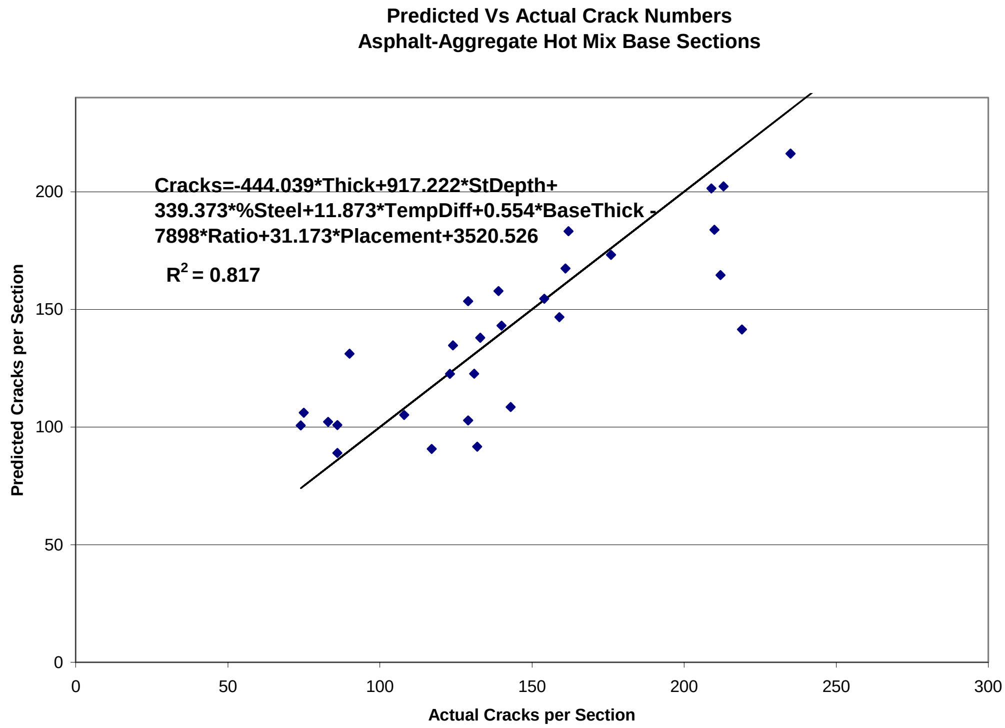

FIGURE 4 ACM sections model.

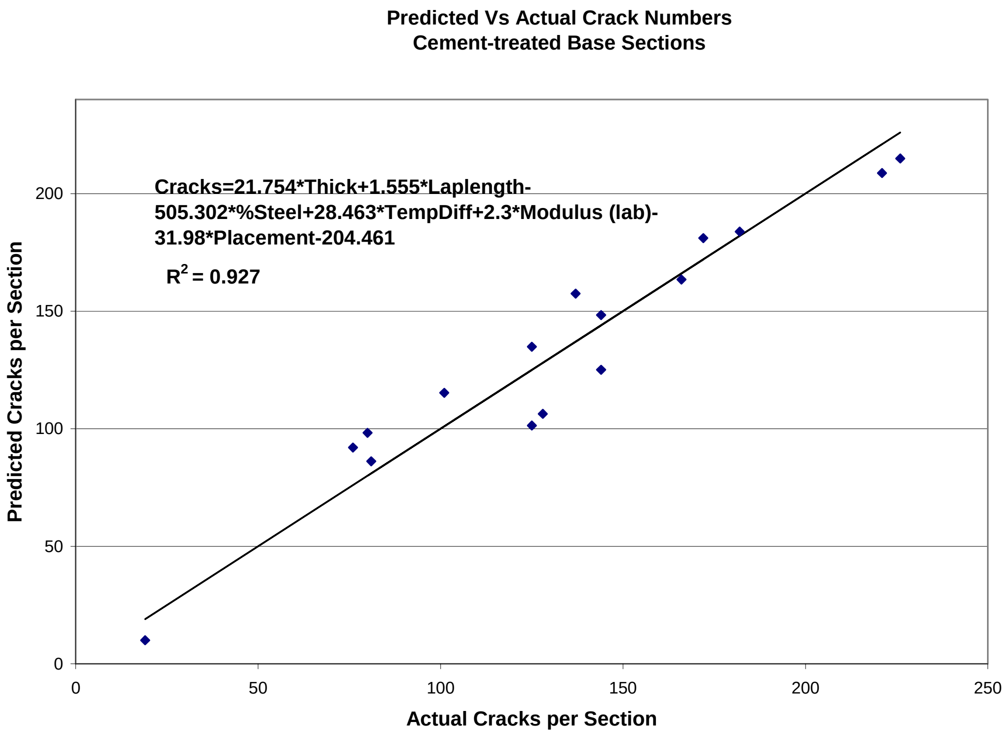

FIGURE 5 CAM sections model.

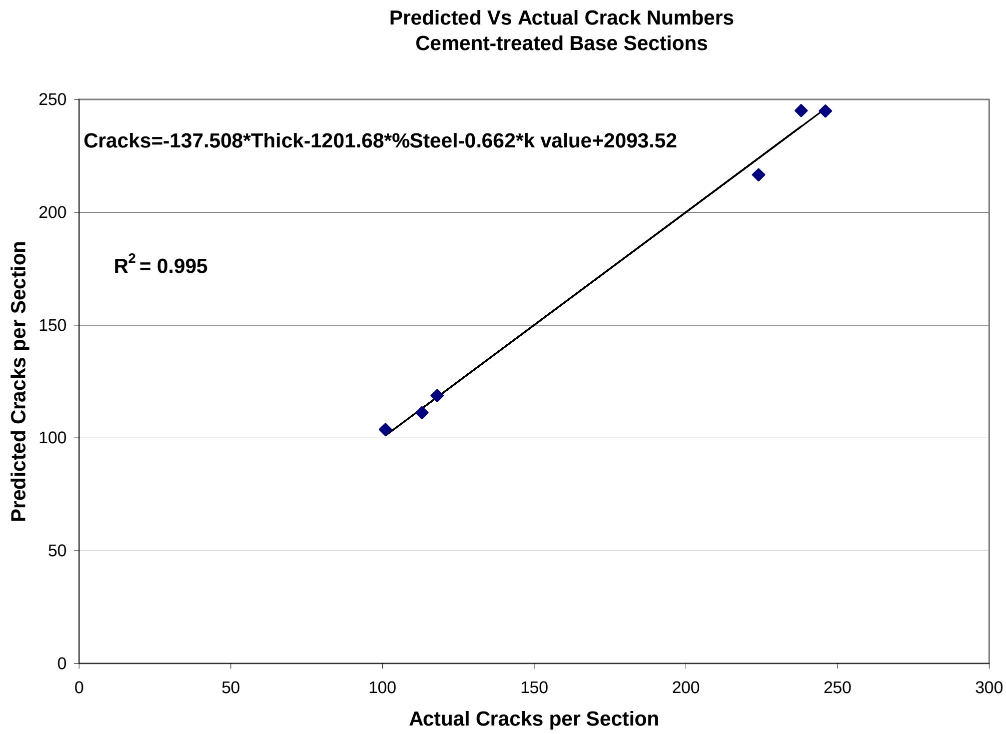

FIGURE 6 Stabilized cement and lime subgrade sections.

apparently contributing to more cracking and an increase in steel content reducing cracking. The soil cement, cement-treated and lime-treated subgrade subgroup showed a mixed response with increasing slab thickness reducing cracking and increasing steel doing the same thing. Obviously, the slab-base bond is directly affecting the early-age cracking patterns generated in the CRC. All factors listed in the LTPP report tables, as well as data taken from the LTPP DataPave online database, were examined for significance and the final models were formulated on a $\alpha<0.05$ for inclusion. Actual slab thickness values from this database were used for analysis as well as lap length and longitudinal and transverse bar diameter.

The steel ratio factor listed in Table 2 is the ratio between steel depth and slab thickness and was included as a factor in all analyses but only met the inclusion criterion for the largest dataset, the ACM sections.

The issue of slab-base bond strength is of importance with respect to the level of stresses generated within the CRC pavement by hydration, steel-concrete bond development, shrinkage and temperature and moisture gradients. The cement-aggregate base forces the generation of relatively closely spaced cracks in the pavement and was specifically adopted for this reason due to performance problems related to widely spaced, open cracks in CRC pavements that result in a reduction in service life under traffic. The high tensile stresses generated by the bonded base, as well as the cracking within the base layer, achieve the goal of reducing crack spacing by exacerbating the temperature response of the pavement section. The statistical analyses of the ACM and CAM sections was only possible after including the temperature difference for the sections as an independent variable. Prior to this point, the independent variables meeting the acceptance criterion of $\alpha<0.05$ could only explain less than half the variance. Interestingly, temperature difference is on the threshold of acceptance into the model for soil cement, cement-treated and lime-treated subgrades at p=0.117 (shown in parenthesis in Table 1), even though the dataset consists of only six sections, implying that it probably is a significant independent variable masked by the lack of data. For all three base types, an increase in temperature difference results in an increase in cracking. Surprisingly, granular bases do not exhibit any marked response to temperature difference (p=0.536) and even show a negative coefficient, indicating that temperature difference affects this base type the least, implying that the majority of the cracking in CRC with unbonded granular bases occurs at early ages. All coefficients and p values in parenthesis were forced into each model to see how close to significance they came but are not considered as part of the models.

The lack of data presents a problem in terms of evaluating the significance of other independent variables in the database. Only the ACM (n=28) and CAM (n=16) subgroups have sufficient data to allow the addition of other statistically significant factors. Interestingly, both these subgroups have placement as a significant contributor to transverse cracking with the only difference between the two being the signs are reversed. Three types of placement techniques were employed in constructing the pavement sections-chairs, mechanical and other. The reversal of the signs between these two groups is consistent with the signs for slab thickness and % steel also being reversed and lends support to the response difference being a real phenomena and not a statistical fluke. This is borne out by lap length also being significant, but only for the CAM sections, even though there was no statistically significant difference between lap lengths in the CAM sections and the others (p = 0.256).

Steel depth is also interesting as a factor as it is only significant for the two sections (Granular and ACM) where % steel correlates with an increase in cracking but is not significant for the stabilized subgrade or cement-treated base sections. The ACM sections had a significantly higher steel depth than all other base types except stabilized bases (p = 0.011) whereas the granular sections had a significantly lower steel depth than the others (p = 0.034). The fact that these two base types represent the extremes of shallow and deep longitudinal steel may explain why this variable was important for them but not the other types. Considering each base type was analyzed separately, this does not explain the lack of significance of steel depth for the stabilized and CAM types as both of these had sufficient range in steel depth values for any significance to become apparent. This is also consistent with the strongly bonded bases changing the interactions of the steel in the cracking process. The strong dependence of cracking response to concrete modulus also argues in favor of this difference as laboratory concrete modulus clearly provides a positive effect on transverse cracking. Both the ACM and stabilized subgroups verge on significance with respect to concrete modulus, and it probably would be significant if more data were available-only 11 of the 28 ACM sections in the analysis had laboratory modulus results. There is a good likelihood that the ACM model fit would be markedly improved with additional concrete modulus data. Again, granular proves the exception as the concrete modulus was not significant in the model, even though all sections used for the analysis had modulus values in the dataset.

Base thickness proved significant only for the ACM sections with none of the other subgroups even approaching the threshold of significance. This may be due to the fact that ACM base thicknesses averaged much lower than any other base type (p = 0.015). Subgrade k values were only important for the stabilized subgrade sections and, probably, for the granular sections, especially considering these values were not measured but were backcalculated from deflection data. It is not surprising that subgrade support is more critical for these two base types but these two base types also had significantly lower k values compared to the ACM and CAM sections (p

= 0.002). Backcalculated base and slab modulus values were also included in the analyses but did not show any significance.

A similar cracking analysis was performed on data from 21 CRC pavements in South Dakota, constructed since 1995. The results are shown in Figure 7. Slab thickness, % steel and steel depth were all significant independent variables in the model (p < .05) with the coefficients having the same signs as those in the LTPP model for granular bases. Other significant factors included pavement width (24 and 26 feet), transverse spacing and size and longitudinal spacing. The analysis was somewhat confounded by multiple design changes occurring at the same time but the results clearly indicate that CRC pavements with granular bases do not respond the same as those with other base types.

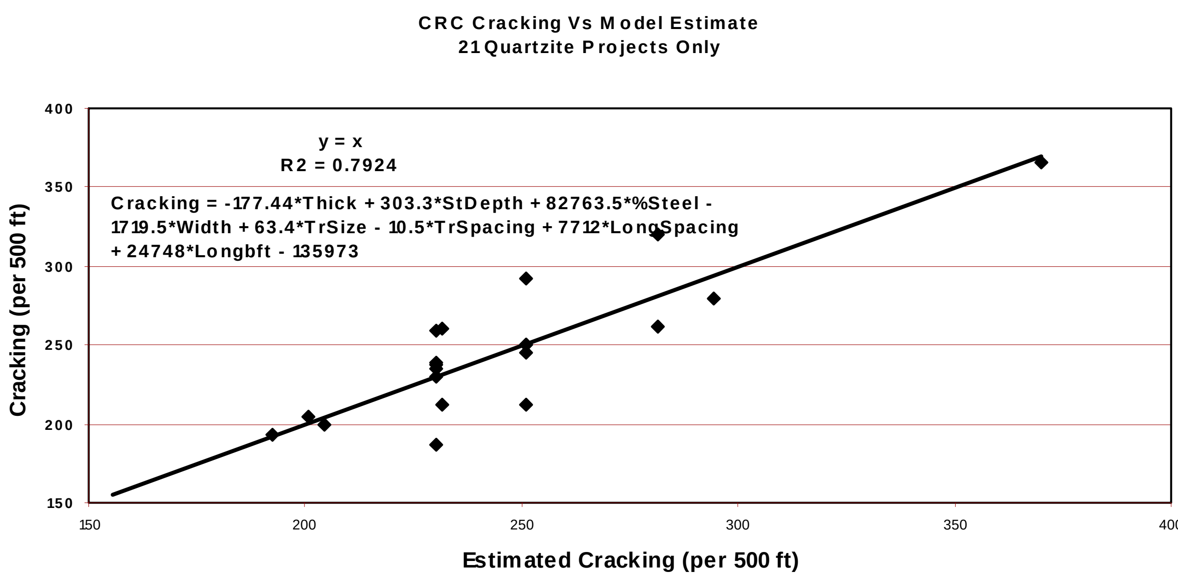

FIGURE 7 South Dakota CRC pavement projects.

DISCUSSION

Although the results of this analysis do not directly address the undesirable cracking occurring on our newer CRC sections, they do provide insight into possible causes. It is interesting to note that, despite the fact that granular base CRC sections exhibit the least cracking of any pavement section type, South Dakota is experiencing severe closely spaced cracking on virtually all our newer CRC pavements, even though the majority of base types are granular. The analysis strongly supports the idea, initially buttressed by the Madison Street I-29 project where larger top size coarse aggregate was used, that the cracking in the newer CRC pavements is being driven by early age stresses that would normally be insufficient to create unacceptable cracking patterns but are acting on the concrete before sufficient concrete-steel bond formation. Both the reduced top size coarse aggregate and the 20% Modified F fly ash in the concrete are slowing bond strength development and fracture resistance and the cracking exhibited in the CRC pavements reflects this by its wandering nature and excessive width. If significant volume change is occurring in recently poured slabs with insufficient restraint due to steel-concrete bond strength being inadequate and base friction not fully developed, the cracks formed are both wide and uncontrolled. The additional presence of both cluster cracking, Y-cracking and network cracking as well as crack spalling all argue in favor of the concrete being unable to resist the early tensile stresses adequately.

A review was made of the numerous changes in materials and design elements for CRC in South Dakota looking for possible means of increasing the resistance to early age cracking. The most obvious and adjustable factor in the entire matrix of changes was the reduction in coarse aggregate top size in the newer CRC compared to the older construction. The seminal work of Abrams(3) on bond strength development versus aggregate size clearly illustrates the pronounced increase in bond strength associated with larger aggregate sizes in the concrete being tested. In addition, a consideration of the energy requirements to achieve concrete fracture from the literature(4) shows an exponential increase in energy at fracture in opposition to aggregate size. Based on these considerations, the nominal top size for coarse aggregate in concrete mix designs for CRC was changed from 1” to 1½”for all new CRC concrete mixes.

An example of this situation is the newly constructed stretch of I-90 eastbound near Exit 55 in Rapid City, SD, which consists of an 11.5” CRC section. Crack mapping was begun soon after construction began and Table 3 below details the results up until the end of October 2004. The progression of spalling and Y-cracking on this project so soon after construction is somewhat alarming, especially considering the coarse aggregate used is limestone which has a low coefficient of thermal expansion and should minimize any thermal contributions to early age stresses. All of these cracks are also extremely wide as the left image in Figure 8 shows and the spalls appear to be due to initial stresses exacerbated under traffic. This pavement had a nominal #6 longitudinal reinforcing depth of 4” due to its thicker section. The image on the right in Figure 8, on the other hand, is from I-29 Madison Street, built the same year using the larger coarse aggregate concrete mixture, and shows a much tighter crack even though the coarse aggregate was an extremely thermally-reactive quartzite, the pavement thickness was 12” and the nominal steel depth was 4” with #7 longitudinal reinforcement. The significant change in cracking behavior with the aggregate size being the only major difference strongly supports going to the coarser mixtures. Also, since greater steel depth contributes to an increase in transverse cracking and crack width, as does increasing steel content, based on the granular model above, another change in current design requirements which could be beneficial with no extra cost involved would be to require a nominal steel depth of 3½” for all new CRC no matter its depth. Reducing the size of the steel to #6 or #5 for new CRC may also be beneficial as this will also increase crack spacing.

TABLE 3 Crack and Damage Progression on I-90 MRM 55 Section

Feature	7/7/2004	8/12/2004	10/29/2004
Number of Cracks	42	58	95
Crack Spacing	15.96	10.91	6.67
Spalls	0	1	49
Y-cracks	0	1	4

FIGURE 8 Relative size comparison of the wider cracks in the I-90 limestone pavement (left) versus the tighter cracks in the I-29, Madison Street pavement (right).

One primary factor which has to be taken into consideration when reviewing existing CRCP design guidelines and practices is the driving force behind many of the recommendations for everything from base type to steel content. The draft mechanistic-empirical design software from the 2002 AASHTO Design Guide was used to model the effects of base type and friction coefficient on punchout performance and cracking. The results of a series of analysis for granular bases with frictional coefficients ranging from 2 to 8 are shown in Figure 9. Table 4 below is taken directly from the design guide and illustrates the ranges and averages for friction coefficients for various base types. The modeling parameters characterized the base as fairly erodable but the modeling results clearly show the profound effect of the friction coefficient on projected performance. To obtain a punchout rating of 10/mile (the default failure threshold) at a reasonable age of 40+ years it was necessary to use a friction coefficient of 8 which is twice the high value for granular base shown in the table. Considering the fact that the first two experimental CRC pavement test sections built in South Dakota are 41 years old and have performed flawlessly with granular bases over their entire lives, the performance basis for the AASHTO Design Guide model may not truly reflect performance issues for South Dakota CRC pavements, especially considering the pavement sections used to validate the model were primarily from Illinois and had cement-treated bases. The design guide does require local calibration and adjustment of the model response to actual performance before implementation.

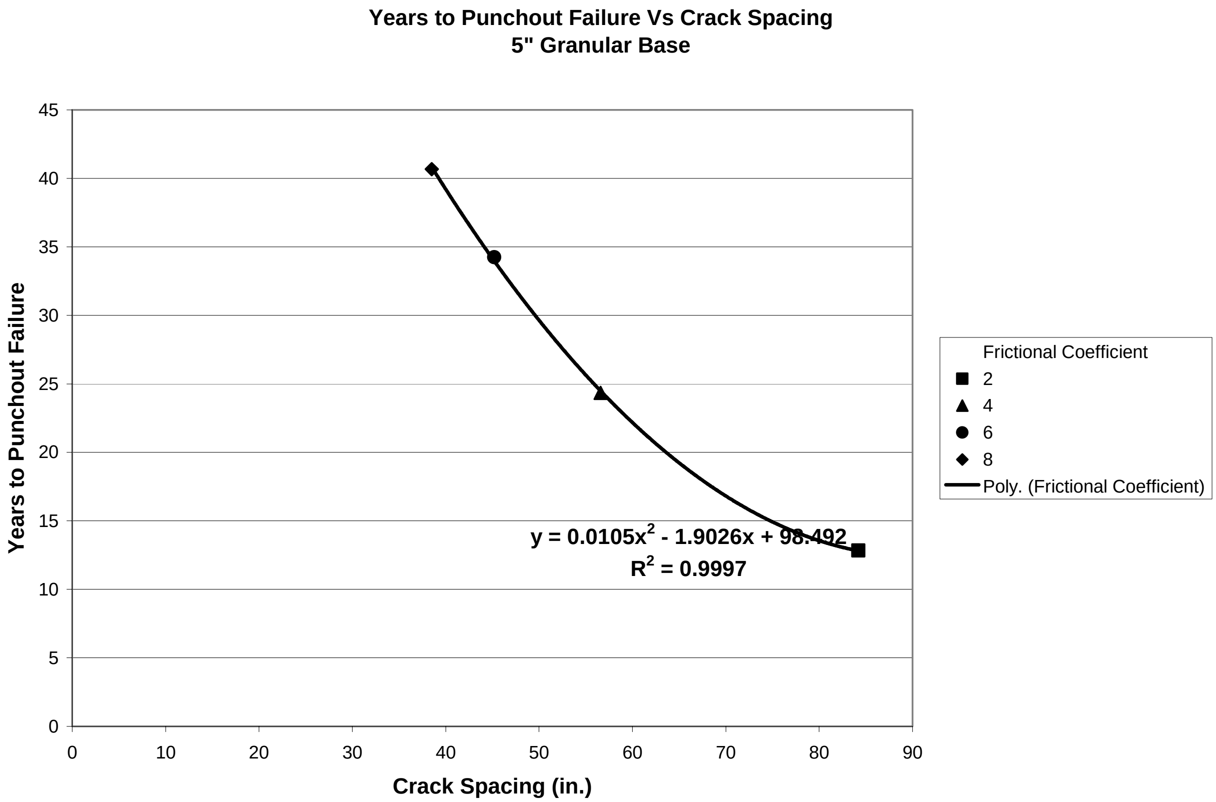

FIGURE 9 Years to punchout failure vs. crack spacing.

TABLE 4 Typical Values of Base/Slab Friction Coefficient Recommended for Design

Subbase/Base type	Friction Coefficient (low – mean – high)
Fine grained soil	0.5 – 1.1 – 2
Sand*	0.5 – 0.8 – 1
Aggregate	0.5 – 2.5 – 4.0
Lime-stabilized clay*	3 – 4.1 – 5.3
ATB	2.5 – 7.5 – 15
CTB	3.5 – 8.9 – 13
Soil cement	6.0 – 7.9 – 23
LCB	3.0 – 8.5 – 20
LCB not cured*	> 36 (higher than LCB cured)

Although much work remains, preliminary surveys on projects and some initial ground penetrating radar (GPR) data indicate that much of the problem with the newer CRC is manifesting shortly after construction is completed. Initial GPR results from two projects on I-90 near Vivian show a tendency for steel depth to decrease toward centerline and show wandering transverse cracks associated directly with transverse steel. In three sections examined using GPR between 70% and 85% of transverse steel had cracks directly over the steel or within a few inches of it. The steel settlement can be ascribed to concrete placement techniques where the greatest concrete load is placed on centerline before being dispersed by the paving operation. A modified chair design with reduced spacing near centerline has been adopted for all new CRC in South Dakota. The association of transverse steel with cracks, although not completely unexpected, is a direct indication of cracking occurring at early ages due to drying shrinkage processes inadequately restrained by steel bond or concrete strength. Cracking directly over steel is not strongly associated with the temperature gradient-induced cracking that relieves stresses in the pavement. This is also consistent with the wandering nature of the cracks, the tendency for network cracking and wide cracks. Transverse bar spacing was included in the LTPP analysis discussed above for all base types and did not correlate with transverse cracking in any model.

CONCLUSIONS AND RECOMMENDATIONS

Since South Dakota is typically seeing very early signs of cracking distress in some new projects, the issue of what has changed and what further steps can be taken to minimize potential problems on future projects is of pressing importance. The results of the LTPP data analysis provide direction on what types of project information is needed to gather to be able to understand contributors to our current undesirable cracking performance. They also give some direct insight into what design detail changes the State might benefit from with respect to next years construction projects. Among these are the following items:

1. South Dakota's current design for nominal steel depth, ranging from 3¼" for 8" thick CRC to 4" for pavements ≥ 11" should be modified to 3½" for all pavements. This will insure minimal crack widths without affecting constructability or performance.
2. The 5" granular base for all new projects should provide an adequate construction platform and sufficient support.
3. The chair support design has been enhanced by reducing the spacing between chairs near centerline to handle the additional loading during paving operations.
4. Several projects should be constructed using 0.6% steel, #5 reinforcement and optimized gradation with a significant 1½" aggregate content to compare crack spacing and width with projects where larger bars and greater steel percentages were used.

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